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SEEPAGE TESTS IN L-D1 BORROW CANAL AT LAKE OKEECHOBEE, FLORIDA

By
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U. S. Geological Survey

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ABSTRACT

Tests were made along Levee D-1 and the adjacent Levee D-1 borrow canal at the west side of Lake Okeechobee near Moore Haven, Florida, to determine the magnitude of the underseepage from the lake. By relating the ground-water gradients in the vicinity to the amount of pickup in a 3½-mile reach of the L-D1 Canal, it was determined that the underseepage factor is about 0.9 cfs per mile per foot of head between the lake and the canal. The coefficient of transmissibility of the underlying materials is about 72,000 gpd/ft. The values are important in the design of a pumping station which will remove increased underseepage when the lake level is raised several feet.

INTRODUCTION

The U.S. Geological Survey, in cooperation with the Central and Southern Florida Flood Control District (C&SFFCD) and the Diston Island Drainage District conducted seepage tests along a 3½-mile reach of the L-D1 Borrow Canal rimming Lake Okeechobee, as shown on figure 1, near Moore Haven, Florida, during the period December 18, 1967 through March 7, 1968. The reach of canal located on the landward side of the Hoover Dike between Culverts No. 1 and 1A, was dug about 1962 to provide fill for raising the elevation of the Hoover Dike and for construction of a seepage dike along the canal's landward side, as shown on figure 2.

The tests were conducted at the request of the C&SFFCD to provide field verification of seepage rates calculated from seepage factors previously determined by Meyer in an investigation of seepage rates along the Hoover Dike during the period 1963-66. In a written communication to C&SFFCD on May 6, 1966, Meyer reported that the seepage factor at Site No. 1, figure 3, was about 1.2 cfs (cubic feet per second) per mile per foot of head between the lake and the canal. In 1963, the U.S. Army Corps of Engineers recommended improvements in outlets and canals in the Nine-Mile Canal area to convey both pumped runoff from the agricultural area and seepage from the lake to a planned pumping station (S-4) located about 4 miles east of Culvert 1A near Clewiston. The Corps of Engineers (1963, plate 9) estimated that the average seepage into the L-D1 Canal from the lake would be about 76 cfs per mile when corresponding levels of the lake and the canal were at 19 and 13 feet, respectively. This represents a seepage factor of about 12.7 cfs per mile per foot of head between the lake and the canal. Because of the wide range between the reported seepage factors it was deemed important that field tests be performed.

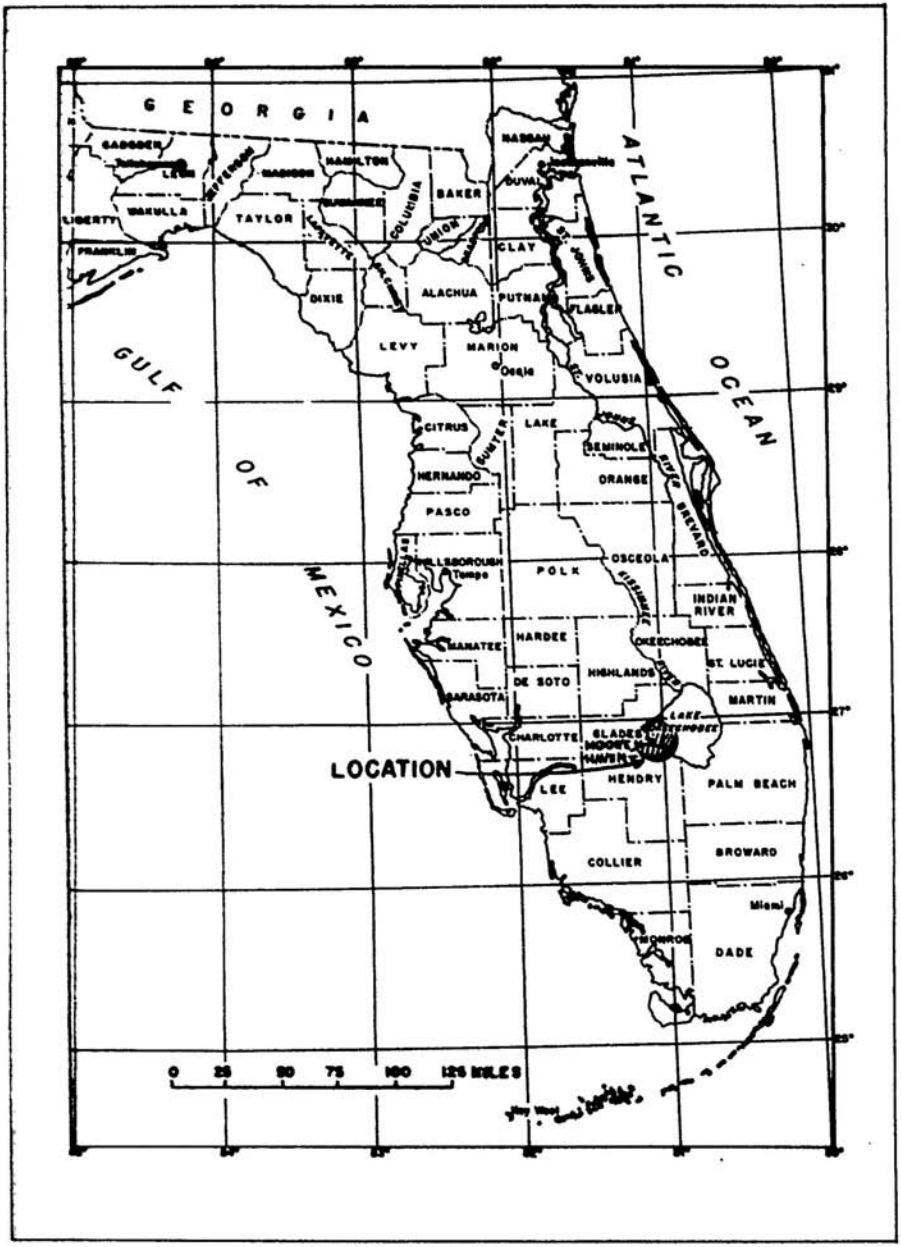

Figure 1. Map of Florida showing area of the investigation.

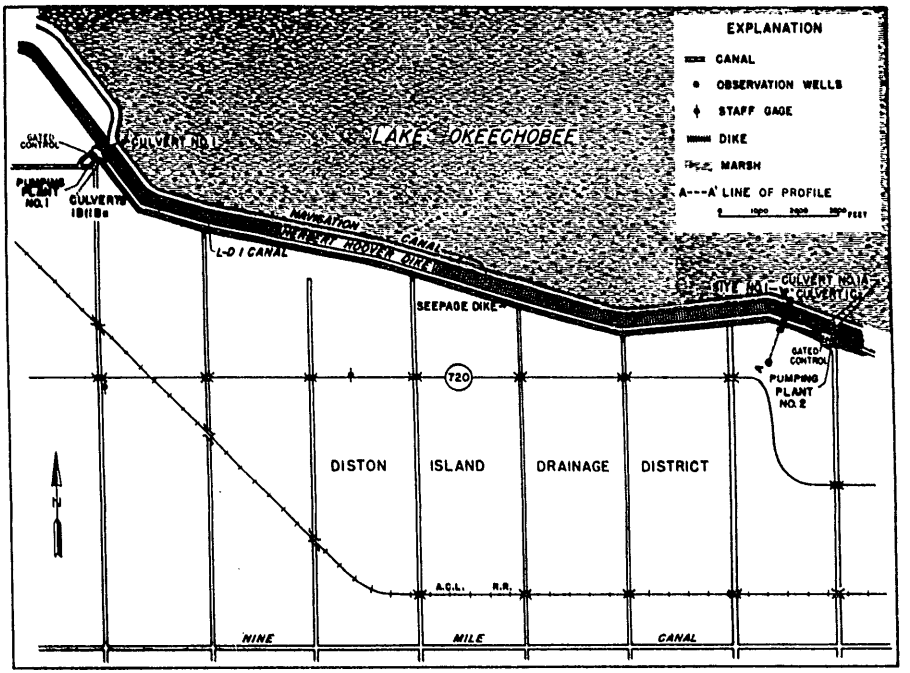

Figure 2. Map of the Test Site.

PURPOSE AND SCOPE

The purpose of the investigation was to evaluate the two reported seepage factors by actually measuring the seepage entering the L-D1 Canal as its level was lowered several feet below that of the lake. The plan was to lower the water level in the canal by both pumped and gravity outflows to the Diston Island Drainage District. The hydraulic gradients to and from the canal were to be determined by utilizing existing observation wells at Site No. 1 (fig. 2). The coefficient of transmissibility would then be computed by relating the measured outflow to the ground-water gradients. The seepage factor would then be computed by relating the total seepage from the lake to the head between the lake and the L-D1 Canal. The findings of this report should be helpful in evaluating seepage rates for future drainage projects and in providing basic information on the methods used to determine seepage factors.

ACKNOWLEDGMENTS

The authors wish to express their appreciation for the cooperation extended by Messrs. Storch, Taylor, Irons, and Lane of the Central and Southern Florida Flood Control District, Messrs. Knecht and Brantley of the Diston Island

Drainage District, Messrs. Koperski and Wiesenfeld of the U.S. Army Corps of Engineers, and Mr. Jensen of Gee and Jensen, Consulting Engineers.

Messrs. Knecht and Springstead of the U.S. Sugar Corp., were instrumental in initiating the tests and provided liaison and helpful suggestions during the investigation.

The work was done under the general supervision of C. S. Conover, District Chief, and the immediate supervision of H. Klein, Subdistrict Chief, of the Water Resources Division, U.S. Geological Survey.

METHODS OF INVESTIGATION

Prior to the tests, a reconnaissance was made of the area and several meetings were held with engineers of the C&SFFCD, the Diston Island Drainage District, and the U.S. Sugar Corp., to arrive at a plan for cooperative assistance during various phases of the tests. Agreements reached were as follows:

1. Personnel of the U.S. Geological Survey would supervise the tests, collect and analyze the data, and prepare a report on the findings. Also, they would replace observation wells 12, 13, and 14 with wells 20, 21, and 22 and install three additional wells (23-25) about 1,200 feet south of the L-D1 Canal in the existing line of wells at Site No. 1.

2. The Diston Island Drainage District would provide pumping facilities and manpower to operate the controls in the culverts and furnish about 40 feet of 6-foot culvert for installation beside Culvert 1B.

3. The U.S. Sugar Corp. would install water-level recording instruments on the observation wells, in the L-D1 Canal, and in Lake Okeechobee.

4. The C&SFFCD would obtain permission and provide the necessary equipment to install the additional culvert, designated herein as Culvert $1B_a$ beside Culvert 1B.

Most of the data was collected during the period January 18 through March 7, 1968. Discharge from the 3½-mile reach of the L-D1 Canal was measured with a current meter in the 6-foot culverts located at each end of the canal. Culvert 1C, with bottom invert at 10.2 feet above mean sea level (msl), was used as a measuring section for determining the gravity flow from the canal, and Culvert $1B_a$, with bottom invert at 5.0 feet above msl, was used to determine the pumped flow. A diesel-powered pump, rated at 125 cfs, was used to lower the water level in the L-D1 Canal several feet below that in the lake. Water-level data were continuously recorded in 14 observation wells, in the L-D1 Canal, and in the lake during the period January 18 through March 7, 1968, as shown on figures 4, 5, 6 and 7. Hydraulic profiles were constructed from water level data for selected days to determine the direction and amounts of seepage into or out of the L-D1 Canal, as shown on figures 8, 9, 10 and 11.

HYDROLOGY

The investigation was divided into three tests in order to determine the seepage under different conditions. The first test comprised an evaluation of the hydrologic conditions during the period December 18 through January 22 when water flowed from the lake into the 3½-mile reach of L-D1 Canal. The second test involved the lowering of the water level in the canal by gravity drainage during the period January 24 through February 2. The third test involved the lowering of the water level in the canal by pumping during the period February 13 through February 20.

The amount of seepage from the lake beneath the 3½-mile length of the Hoover Dike depends primarily on the coefficients of transmissibility of the aquifers and the hydraulic gradients. The transmissibility is assumed to be uniform along the reach of dike because the geologic section prepared by the Corps of Engineers (1961, plate 13) shows that the sub-surface materials at Site No. 1 are generally similar to those underlying the dike from Culvert 1B to Culvert 1C.

The transmissibility (T) is defined as the amount of water, in gallons per day, at the prevailing water temperature, that would pass through a 1-foot wide section of the full saturated thickness of the aquifer under a unit hydraulic gradient and is determined from the equation

$$T = \frac{Q}{IL} \tag{1}$$

where Q is the seepage in gallons per day, L is the length of canal affecting the seepage, in feet, and I is the average hydraulic gradient. The gradient (I) is determined by the equation

$$I = \frac{h}{d} \tag{2}$$

where h is the head, in feet, between the water levels in two observation wells in the same aquifer at Site No. 1 and d is the distance, in feet, between the wells.

It is assumed that T and L are constant, that flow is steady-state, and that the hydrologic influence of filtercakes in the lakeside navigation canal and in the L-D1 Canal are uniform. The discharge Q_m flowing into or out of the L-D1 Canal is related to the seepage by the equation

$$Q_m = Q_l + Q_d + \Delta S_m \tag{3}$$

where Q_l is the seepage from or into the lake, Q_d is the seepage from or into the Diston Island Drainage District, and ΔS_m is the change in storage in the L-D1 Canal expressed in terms of daily mean discharge. Elements Q_m, Q_l, and Q_d are positive when the direction of seepage is toward the L-D1 Canal and the discharge is from the canal. ΔS_m is positive when the water level in the L-D1 Canal is falling.

Equations 4 and 5 below express the seepages related to the lake and Diston Island Drainage District in terms of the coefficient of transmissibility and the hydraulic gradients (see equation 1) where I_l is the hydraulic gradient related to the lake and I_d is that related to the drainage district.

$$Q_l = TLI_l \tag{4}$$

$$Q_d = TLI_d \tag{5}$$

Equation 6 below is obtained by substituting equations 4 and 5 in equation 3 and equation 7 is obtained by solving equation 6 in terms of T.

$$Q_m = TL\,(I_l + I_d) + \Delta S_m \tag{6}$$

$$T = \frac{Q_m - \Delta S_m}{L\,(I_l + I_d)} \tag{7}$$

The seepage factor (S_e) is defined herein as the rate of seepage per mile length of recharge section per foot of head between the recharge boundary and the discharge boundary. It is determined by using the equation

$$S_e = \frac{Q}{L(h_1 - h_2)} \tag{8}$$

where Q is the seepage rate (cfs), L is the length of the recharge section (miles), h_1 is the elevation of the water level at the recharge boundary (feet), and h_2 is the elevation of the water level at the discharge boundary (feet).

Figure 3 is a profile across the Hoover Dike at Site No. 1 showing the aquifers and confining beds. Hydrologic units A-1,. A-2, and A-3 are designated as aquifers and hydrologic units C-1, C-2 and C-3 are designated as confining beds. Some seepage, however, occurs through each of these beds but most seepage occurs through beds A-1, A-2, and A-3 because they are more permeable and are located close to sources of recharge (the Lake Okeechobee Navigation Canal) and discharge (the L-D1 Canal).

Bed A-1 is chiefly a sandy, marly limestone whose upper surface is case-hardened. Many small solution holes account for zones of high permeability in A-1. Bed A-2 is composed mostly of shells and is highly permeable. Bed A-3 is composed of sand and sandstone and is moderately permeable.

Bed C-1 is composed of black organic muck and is relatively impermeable. Its confining ability is locally ineffective where it is transected by many canals and ditches. Bed C-2 is composed of sand and is only slightly less permeable than beds A-1, A-2, and A-3. Its confining ability is locally ineffective where it is transected by the deep borrow canals. Bed C-3 is composed of green clay and is relatively impermeable. Its confining ability is very effective because it has low permeability and is not breached by the borrow canals. Bed C-3 retards the

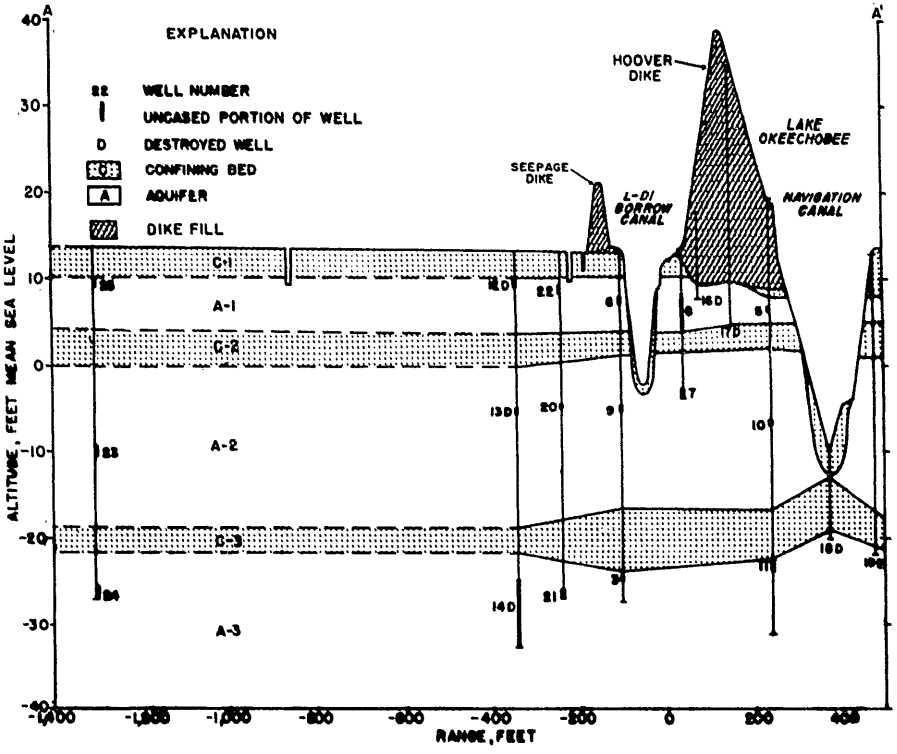

Figure 3. Profile along line A-A′ at site no. 1 (Station 180+00, Levee D1) showing aquifers and confining beds.

movement of water into and out of bed A-3; therefore seepage through bed A-3 is considered negligible. Most of the seepage beneath the dike at Site No. 1 is considered to occur through the upper 30 feet of material.

Of equal importance in the analysis of underseepage is the role played by silt deposits lining the sides and bottoms of the borrow canals. These deposits were formed chiefly by the settling-out of the fine fractions from the excavated material and by the accumulation of organic sediments derived from dead vegetation and from erosion of the surface materials.

Because the level of the lake is usually higher than the water level in the Diston Island Drainage District, hydrostatic pressure has caused a filtercake to form on the bottom and walls of the lakeside canal. The buildup of the filtercake has probably caused a progressive reduction in the seepage from the lake over a period of years. The loss in head across the filtercake is an important factor in analyzing aquifer coefficients because more head is required to move water at a given rate through the filtercake than to move water at the same rate through a like thickness of aquifer. Therefore, the determination of aquifer coefficients is related to hydraulic gradients within the aquifer itself, and not to gradients influenced by the filtercake.

Figures 4–7 are graphs of fluctuations in water levels at Site No. 1 during the period January 18–March 7, 1968. Figure 4 is a comparison of the fluctuations in water levels in Lake Okeechobee with those in the L-D1 Canal. The maximum head attained between the lake and the canal was about 5 feet. Figures 5–7 show the fluctuations in water levels in observation wells tapping the three aquifers at Site No. 1. The lines representing fluctuations in the wells are coded by a number of dots so that the line with the least dots represents the well site nearest to the lake while the line with the most dots represents the well site farthest from the lake. Water levels in most wells were affected by changes in the stage of the L-D1 Canal.

A comparison of figure 5 with figure 6 shows that water-level fluctuations in aquifers A-1 and A-2 at comparable distances from the canal were essentially identical with respect to time and amplitude. Thus both aquifers are hydraulically connected to the L-D1 Canal and to each other, and therefore function locally as a single hydrologic unit. A comparison of figure 7 with figures 5 and 6 shows that water-level fluctuations in aquifer A-3 were less affected by changes in the stage of the L-D1 Canal. The reduction in the amplitude of the fluctuations is caused by the confining effect of the overlying bed C-3. Water levels in wells 23, 24, and 25, located about 1,200 feet south of the L-D1 Canal (fig. 3), were unaffected by changes in the stage of the canal during the tests due to the effect of constant head in a field ditch located about 450 feet north of the wells.

TEST 1

During the period January 18 through January 22, 1968 the water level in the L-D1 Canal was generally less than 0.2 foot below that of Lake Okeechobee because of uncontrolled inflow of water from the lake (fig. 4). Lake water flowed directly into the L-D1 Canal through Culvert 1B (fig. 2) because debris had lodged in the automatic flap gate and prevented its closure. Normally, the gate at Culvert 1B would close by differential head produced by higher water levels in the lake and in the lakeside bay of Diston Island's Pumping Plant No. 1. During this period Culvert 1C was closed. This condition was observed to have been in effect since December 18, 1967.

If no seepage occurred from the L-D1 Canal, the water level in the canal would have reached equilibrium with that of the lake and flow into the canal would have ceased. The lower head in the canal, however, indicated that the flow entering the canal through Culvert 1B was roughly equivalent to the total amount of seepage from the canal into the Diston Island Drainage District along its 3½-mile reach.

Five measurements of the flow into the canal ranged from 14 to 47 cfs during the period December 18 through January 22. At times the velocity of flow varied greatly within a single measurement and once reverse flow was observed. These variations in flow were caused by head changes attributed to seiche of the

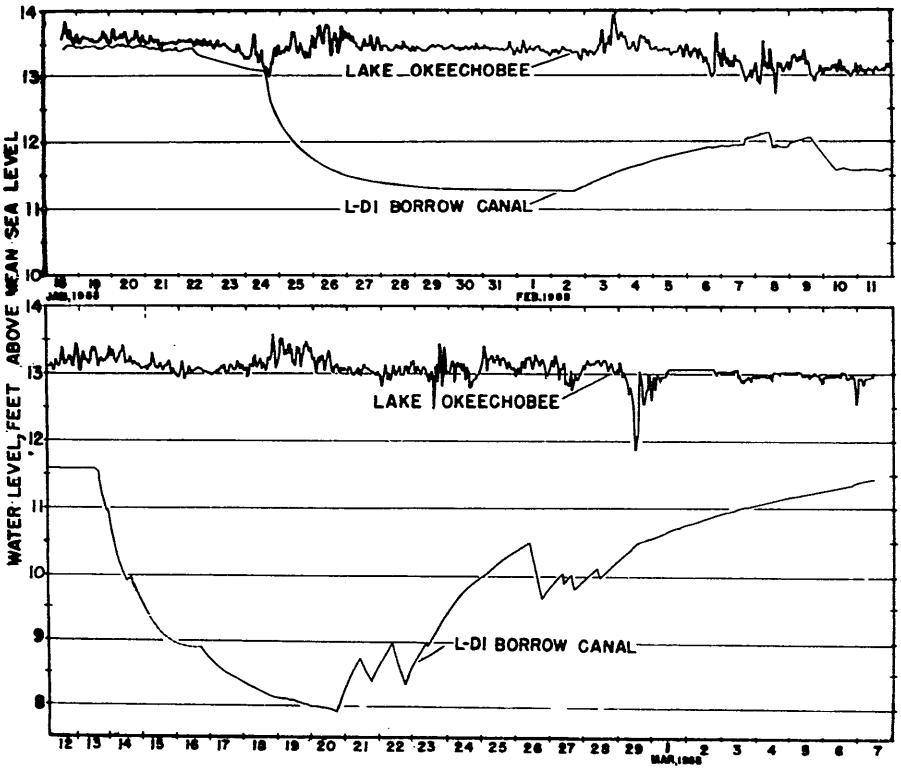

Figure 4. Graphs of water levels in Lake Okeechobee and in the L-D1 Borrow Canal, January 18–March 7, 1968.

lake. The seepage factor and coefficient of transmissibility obtained from this test is considered less accurate than those obtained from tests 2 and 3 because head differences were better known and held more constant during the latter tests.

SEEPAGE ANALYSIS

Hydraulic profiles were constructed to show the distribution of water levels in the subsurface materials at Site No. 1 at times when the flow into the L-D1 Canal was measured at Culvert 1B. Simultaneous water-level data at the site were obtained by measuring the depth to water below a point of known elevation in each well. The water level was then related to mean sea level datum.

The amount of seepage from the canal into the Diston Island Drainage District depends primarily on the average coefficients of transmissibility of the aquifers and the hydraulic gradients in the aquifers along the 3½-mile length of the canal.

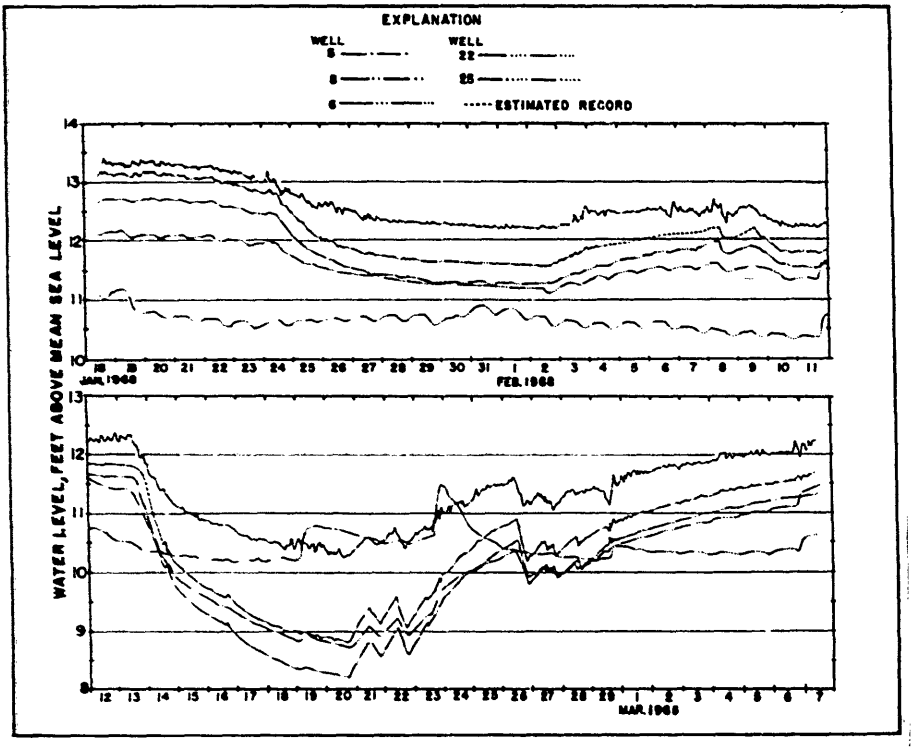

Figure 5. Graphs of water levels in wells 5, 8, 6, 22, and 25 in aquifer No. 1, January 18–March 7, 1968.

Figure 8 is a hydraulic profile showing the distribution of heads and equipotential lines on January 3, 1968. The flow through Culvert 1B was 14.7 cfs and 13.9 cfs at 11:45 a.m. and 2:30 p.m., respectively. The average flow was assumed to be 14.3 cfs, or 9.24 mgd (million gallons per day). It was also assumed that the water level in the canal did not change.

By estimating the location of a ground-water divide beneath the Hoover Dike (fig. 8) it was determined that 14.3 cfs flowed from the L-D1 Canal into the district through A, the upper 90 percent of saturated thickness while 1.6 cfs flowed from the lake through B, the lower 10 percent. Thus, the total seepage (Q) into the district was 15.9 cfs, or about 10.3 mgd. The hydraulic gradient (I) was computed by averaging the head (h) between wells 6 and 22 and between wells 9 and 20, by measuring the distance (d) between the wells and by solving equation 2 (p. 5) as follows:

$$I = \frac{\text{Average h}}{d} = \frac{0.56 \text{ foot}}{138 \text{ feet}}$$

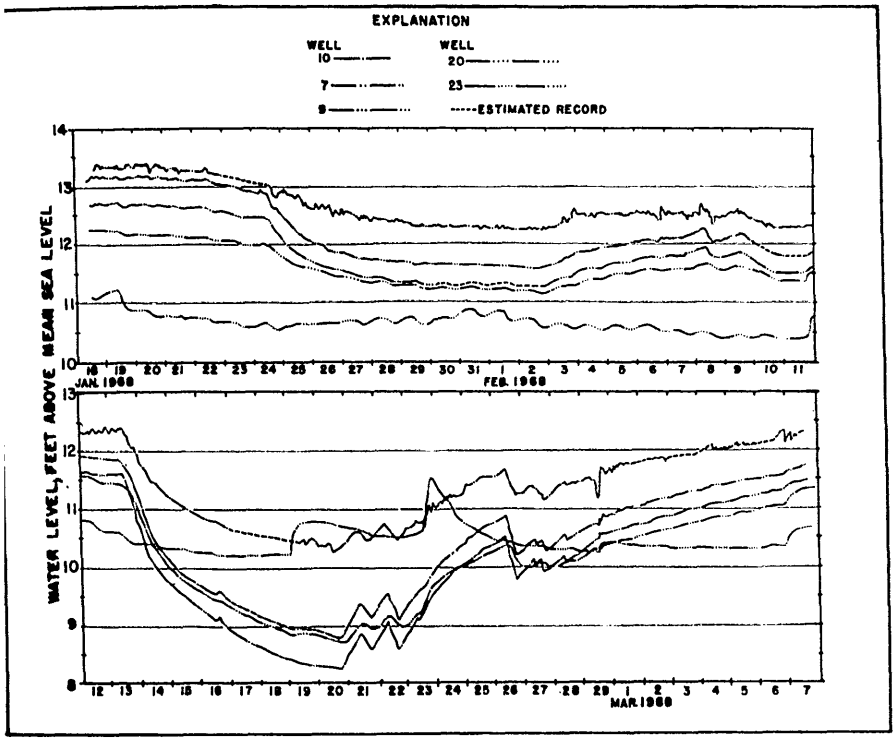

Figure 6. Graphs of water levels in wells 10, 7, 9, 20, and 23 in aquifer No. 2, January 18–March 7, 1968.

The coefficient of transmissibility (T) was computed by substituting the values of I, Q, and L in equation 1 and solving as follows:

$$T = \frac{10{,}300{,}000 \text{ gpd}}{\dfrac{(.56 \text{ ft.}) \times (18{,}810 \text{ ft.})}{(138 \text{ ft.})}} = 135{,}000 \text{ gpd/ft.}$$

The seepage factor (S_e) was computed by solving equation 8, and by assuming that the lake was the recharge boundary and well 23 was the discharge boundary:

$$S_e = \frac{15.9 \text{ cfs}}{3.56 \text{ mi.} \times (13.69 - 11.15) \text{ ft}} = 1.8 \text{ cfs/mi/ft}$$

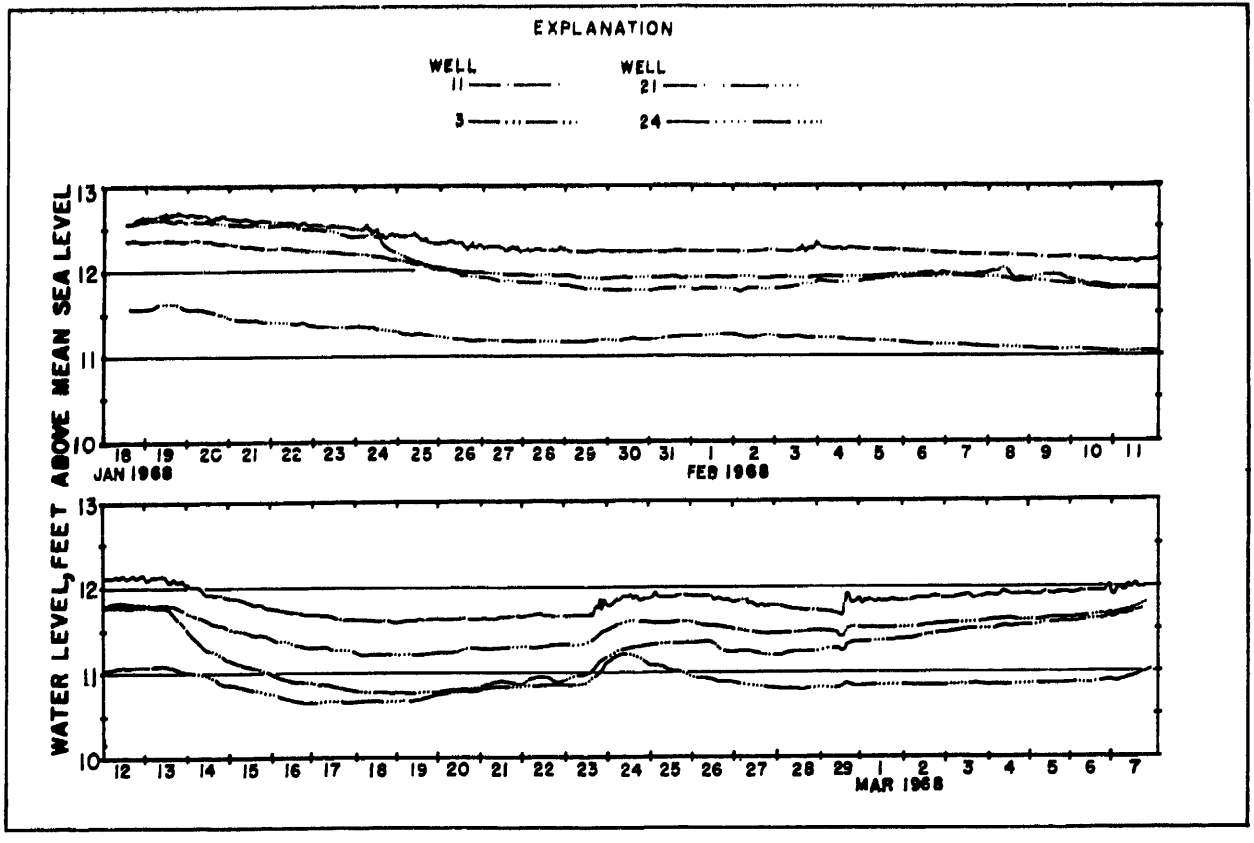

Figure 7. Graphs of water levels in wells 11, 3, 21, and 24 in Aquifer No. 3, January 18-March 7, 1968.

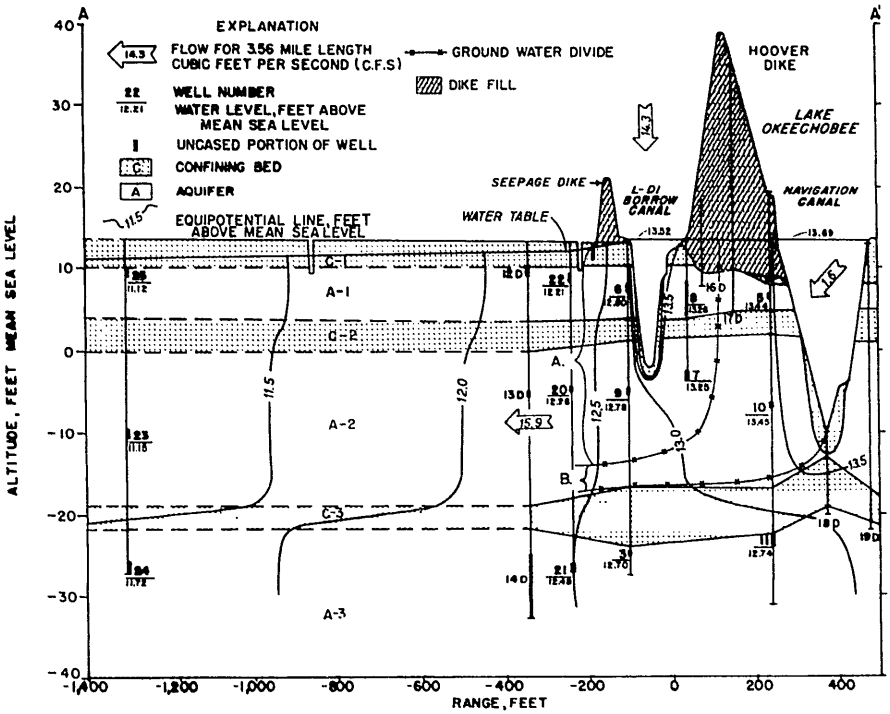

Figure 8. Hydraulic profile along line A-A′ at Site No. 1, January 3, 1968.

The solution of equation 8 yields a seepage factor of 1.8 cfs per mile per foot of head between the lake and well 23. This value exceeds the previous estimated value of 1.2 by about 50 percent. Part of this difference was due to storage changes in the canal caused by seiche, and part was due to a nonsteady state of flow from the canal caused by changing head in the canal and to nonsteady ground-water levels.

TEST 2

The objective of the second test was to determine the amount of seepage from the lake when the level of the L-D1 Canal was lowered by gravity flow. On January 22 the debris in Culvert 1B (fig. 2) was removed and the automatic flap gate closed. After the gate was closed, the reach of canal became ponded and the water level declined toward a static level between the water level in the lake and that in the Diston Island Drainage District (figs. 4–7). On January 24, before equilibrium was reached, Culvert 1C (fig. 2) was opened and the level of the canal rapidly declined (fig. 4). During the period January 25 through February 2, the discharge measured through Culvert 1C was as follows:

Date	Time	Discharge (cfs)
January 25	8:10 a.m.	16
January 26	12:00 p.m.	14
January 29	10:00 a.m.	8.3
February 2	10:45 a.m.	7.2

Water levels in the L-D1 Canal (fig. 4) and in observation wells (figs. 5–7) had nearly stabilized after about a week of gravity flow indicating that a steady-state flow condition had been essentially attained.

SEEPAGE ANALYSIS

The hydraulic gradients at Site No. 1 on January 29, 1968, as shown on figure 9, just prior to steady-state conditions, were analyzed using the aforementioned equations.

The hydraulic gradient I_l between wells 10 and 7 was assumed to reflect the seepage from the lake into the L-D1 Canal; and the hydraulic gradient I_d between wells 9 and 20 was assumed to reflect the seepage loss from the L-D1 Canal to the Diston Island Drainage District. I_l and I_d were computed using equation 2.

$$I_l = \frac{12.30 - 11.65 \text{ ft}}{203 \text{ ft}} = \frac{0.65 \text{ ft}}{203 \text{ ft}} = 0.00320$$

$$I_d = \frac{11.31 - 11.35 \text{ ft}}{138 \text{ ft}} = \frac{-0.04 \text{ ft}}{138 \text{ ft}} = 0.00029$$

The coefficient of transmissibility was computed using equation 7. The change in canal storage, ΔS_m, was determined to be equivalent to 0.5 cfs; Q_m is 8.3 cfs.

$$T = \frac{Q_m - \Delta S_m}{L(I_l + I_d)} = \frac{8.3 \text{ cfs} - 0.5 \text{ cfs}}{18{,}810 \text{ ft } (0.00320 + (-0.00029))}$$

$$= 0.1425 \text{ cfs/ft} \times 0.64632 \text{ mgd/cfs}$$

$$= 0.09210 \text{ mgd/ft} = 92{,}100 \text{ gpd/ft}$$

The seepage from the lake beneath the Hoover Dike, Q_l was computed using equation 4; and the seepage loss from the L-D1 Canal to the district, Q_d was computed using equation 5.

$$Q_l = T \times L \times I_l = 92{,}100 \text{ gpd/ft} \times 18{,}810 \text{ ft} \times 0.00320$$

$$= 5.544 \text{ mgd} \times 1.5472 \text{ cfs/mgd}$$

$$= 8.578 \text{ cfs} = 8.6 \text{ cfs}$$

$$Q_d = T \times L \times I_d = 92{,}100 \text{ gpd/ft} \times 18{,}810 \text{ ft} \times (-0.00029)$$

$$= 0.5024 \text{ mgd} \times 1.5472 \text{ cfs/mgd}$$

$$= 0.777 \text{ cfs} = -0.8 \text{ cfs}$$

The analysis was checked using equation 3.

$$Q_m = Q_l + Q_d + \Delta S_m$$

$$8.3 \text{ cfs} = 8.6 \text{ cfs} + (-0.8 \text{ cfs}) + 0.5 \text{ cfs} = 8.3 \text{ cfs}$$

The seepage factor, S_e, was computed using equation 8. L is expressed in miles, h_1 is the elevation of the water level in the lake, and h_2 is the elevation of the water level in the L-D1 Canal.

$$S_e = \frac{Q_l}{L(h_1 - h_2)} = \frac{8.6 \text{ cfs}}{3.56 \text{ mi } (13.45 - 11.31) \text{ ft.}}$$

$$= \frac{8.6 \text{ cfs}}{3.56 \text{ mi } (2.14 \text{ ft})} = 1.1 \frac{\text{cfs/mi}}{\text{ft}}$$

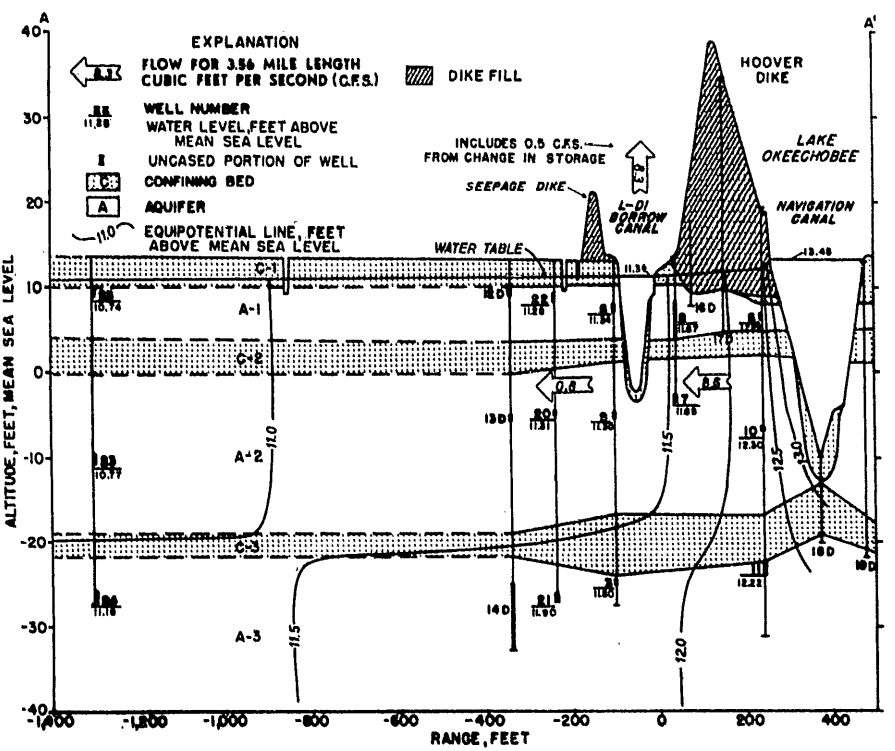

Figure 9. Hydraulic profile along line A-A′ at Site No. 1, January 29, 1968.

The results indicate that: (l) T equals 92,100 gpd/ft, (2) 8.6 cfs seeped from the lake beneath the Hoover Dike, (3) of the 8.3 cfs measured as discharge from the L-D1 Canal, 7.8 cfs seeped from the lake and 0.5 cfs was related to depletion of storage in the canal, (4) 0.8 cfs seeped from the L-D1 Canal into the Diston Island Drainage District, and (5) the seepage factor was 1.1 cfs per mile per foot of head between the lake and the L-D1 Canal.

The aforementioned analysis, however, did not compensate for higher water levels in the drainage district south of Pumping Plant No. 1. During the test period, Pumping Plant No. 1 was operating and the water level in the canal south of the pump was held at about 14 feet above msl. Lacking specific water-level data in the district between Site No. 1 and Pump Plant No. 1, the hydraulic gradient between wells 9 and 20, located south of the canal at Site No. 1 (fig. 9) was assumed to represent the seepage along one-third of the length of the canal between Culverts 1B and 1C. A reversed hydraulic gradient equal to that between wells 9 and 20 was assumed to represent the seepage along the remaining length. Therefore, seepage along one-third of the distance was from the canal into the district while seepage along the remaining distance was from the district to the L-D1 Canal. Thus the net result was seepage from the district into the canal along one-third its length.

The data were reanalyzed using equations 2–8, and the same wells for hydraulic gradients. However, in equation 5 the element Q_d is representative of the net seepage along the length L as shown in the following equation:

$$Q_d = T \times I_d \times 2/3\ L - T \times I_d \times 1/3\ L$$
$$= T \times I_d\ (2/3\ L - 1/3\ L)$$
$$= \frac{T \times I_d \times L}{3}$$

The coefficient of transmissibility was computed by substituting $I_d/3$ in equation 7.

$$T = \frac{Q_m - \Delta S_m}{L\,(I_l + 1/3\ I_d)} = \frac{8.3\ \text{cfs} - 0.5\ \text{cfs}}{18{,}810\ \text{ft}\left(0.00320 + \frac{0.00029}{3}\right)}$$

$$= 0.1258\ \text{cfs/ft} \times 0.64632\ \text{mgd/cfs}$$

$$= 0.08131\ \text{mgd/ft} = 81{,}300\ \text{gpd/ft}$$

The seepage from the lake, Q_l, was computed using equation 4; and the net seepage from the Diston Island Drainage District to the L-D1 Canal, Q_d, was computed using a modification of equation 5.

$$Q_l = T \times L \times I_l = 81{,}300 \text{ gpd/ft} \times 18{,}810 \text{ ft} \times 0.00320$$
$$= 4.894 \text{ mgd} \times 1.5472 \text{ cfs/mgd}$$
$$= 7.57 \text{ cfs} = 7.6 \text{ cfs}$$
$$Q_d = \frac{T \times L \times I_d}{3} = \frac{81{,}300 \text{ gpd/ft} \times 18{,}810 \text{ ft} \times 0.00029}{3}$$
$$= 0.1483 \text{ mgd} \times 1.5472 \text{ cfs/mgd}$$
$$= 0.229 \text{ cfs} = 0.2 \text{ cfs}$$

The analysis was checked using equation 3.

$$Q_m = Q_l + Q_d + \Delta S_m$$

$$8.3 \text{ cfs} = 7.6 \text{ cfs} + 0.2 \text{ cfs} + 0.5 \text{ cfs} = 8.3 \text{ cfs}$$

The seepage factor, S_e, was computed using equation 8.

$$S_e = \frac{Q_l}{L(h_1 - h_2)} = \frac{7.6 \text{ cfs}}{3.56 \text{ mi } (13.45 - 11.31) \text{ ft}}$$
$$= \frac{7.6}{3.56 \text{ mi } (2.14 \text{ ft})} = 1.0 \frac{\text{cfs/mi}}{\text{ft}}$$

The results of the reanalysis indicate that: (1) T equals 81,300 gpd/ft, (2) 7.6 cfs seeped from the lake beneath the Hoover Dike, (3) of the 8.3 cfs measured as discharge from the L-D1 Canal, 7.6 cfs seeped from the lake, 0.2 cfs seeped from the district, and 0.5 cfs was related to depletion of storage in the canal, and (4) the seepage factor was 1.0 cfs per mile per foot of head between the lake and the L-D1 Canal.

On February 2, the flow in the canal appeared to reach a steady-state condition and analysis was made of the hydraulic profile at Site No. 1 (fig. 10).

Again, wells 10 and 7 were used to determine the hydraulic gradient I_l and wells 9 and 20 were used to determine the hydraulic gradient I_d.

$$I_l = \frac{12.24 - 11.58 \text{ ft}}{203 \text{ ft}} = \frac{0.66 \text{ ft}}{203 \text{ ft}} = 0.00325$$

$$I_d = \frac{11.22 - 11.28 \text{ ft}}{138 \text{ ft}} = \frac{-0.06 \text{ ft}}{138 \text{ ft}} = -0.00043$$

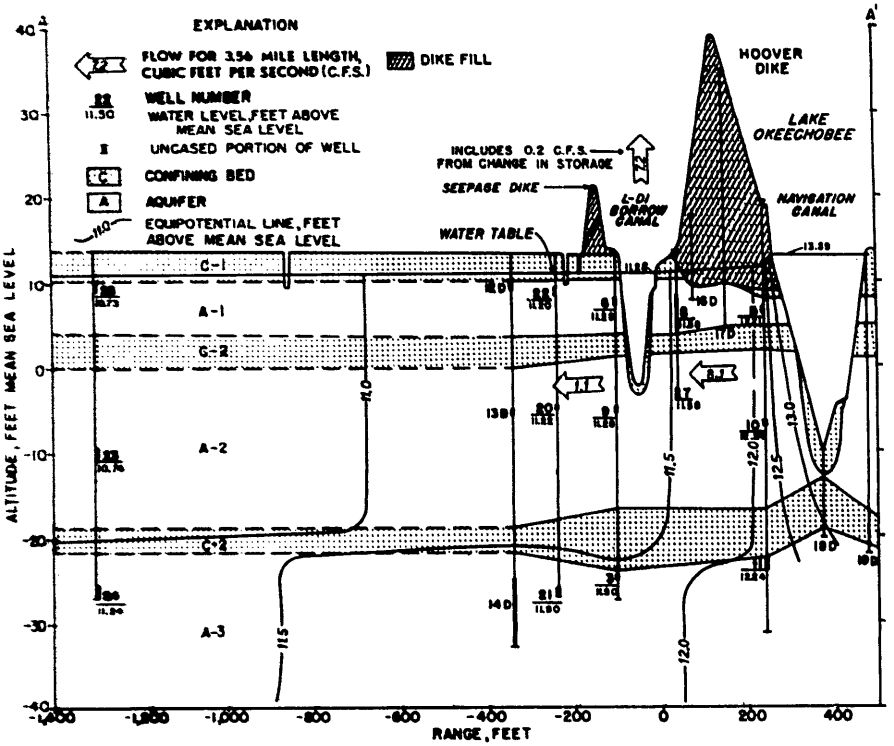

Figure 10. Hydraulic profile along line A-A′ at Site No. 1, February 2, 1968.

The coefficient of transmissibility was computed using equation 7. The change in canal storage, ΔS_m, was determined to be equivalent to 0.2 cfs; Q_m was 7.2 cfs.

$$T = \frac{Q_m - \Delta S_m}{L\,(I_l + I_d)} = \frac{7.2 - 0.2 \text{ cfs}}{18{,}810 \text{ ft } (0.00325 + (-0.00043))}$$

$$= 0.1320 \text{ cfs/ft} \times 0.64632 \text{ mgd/cfs}$$

$$= 0.08530 \text{ mgd/ft} = 85{,}300 \text{ gpd/ft}$$

The seepages from the lake, Q_l, and into the Diston Island Drainage District, Q_d, were computed using equations 4 and 5.

$$Q_l = T \times L \times I_l = 85{,}300 \text{ gpd/ft} \times 18{,}810 \text{ ft} \times 0.00325$$

$$= 5.215 \text{ mgd} \times 1.5472 \text{ cfs/mgd}$$

$$= 8.07 \text{ cfs} = 8.1 \text{ cfs}$$

$$Q_d = T \times L \times I_d = 85{,}300 \text{ gpd/ft} \times 18{,}810 \text{ ft} \times -0.00043$$

$$= 0.6899 \text{ mgd} \times 1.5472 \text{ cfs/mgd}$$

$$= -1.07 \text{ cfs} = -1.1 \text{ cfs}$$

The analysis was checked using equation 3.

$$Q_m = Q_l + Q_d + \Delta S_m$$

$$7.2 \text{ cfs} = 8.1 \text{ cfs} + (-1.1 \text{ cfs}) + 0.2 \text{ cfs} = 7.2 \text{ cfs}$$

S_e was computed using equation 8.

$$S_e = \frac{Q_l}{L\,(h_1 - h_2)} = \frac{8.1 \text{ cfs}}{3.56 \text{ mi } (13.39 - 11.26) \text{ ft}}$$

$$= \frac{8.1 \text{ cfs}}{3.56 \text{ mi} \times 2.13 \text{ ft}} = 1.07 \frac{\text{cfs/mi}}{\text{ft}}$$

$$= 1.1 \frac{\text{cfs/mi}}{\text{ft}}$$

The results indicate that: (1) T equals 85,300 gpd/ft, (2) 8.1 cfs seeped from the lake beneath the Hoover Dike, (3) of the 7.2 cfs measured as discharge from the L-D1 Canal, 7.0 cfs seeped from the lake and 0.2 cfs was related to depletion of storage in the canal, (4) 1.1 cfs seeped from the L-D1 Canal into the district, and (5) the seepage factor was 1.1 cfs per mile per foot of head between the lake and the L-D1 Canal.

Again the data were reanalyzed to compensate for higher water levels in the western part of the district. The hydraulic gradients between wells 10 and 7 and between wells 9 and 20 were used to compute I_l and I_d. As previously described, the gradient between wells 9 and 20 was assumed to be representative along one-third of the length of canal and reversed equivalent gradient was assumed to be representative along the remaining two-thirds of the length of canal.

The coefficient of transmissibility was computed using the following equation: ΔS_m was determined to be equivalent to 0.2 cfs; Q_m was 7.2 cfs.

$$T = \frac{Q_m - \Delta S_m}{L\left(I_l + \frac{I_d}{3}\right)} = \frac{7.2 - 0.2 \text{ cfs}}{18{,}810 \text{ ft}\left(0.00325 + \frac{0.00043}{3}\right)}$$

$$= \frac{7.0 \text{ cfs}}{18{,}810 \text{ ft } (0.003393)} = 0.1097 \text{ cfs/ft}$$

$$= 0.1097 \text{ cfs/ft} \times 0.64632 \text{ mgd/cfs}$$

$$= 0.07090 \text{ mgd/ft} - 70{,}900 \text{ gpd/ft}$$

The seepages from the lake, Q_l, and from the Diston Island Drainage District, Q_d, were computed using equations 4 and 5.

$$Q_l = T \times L \times I_l = 70{,}900 \text{ gpd/ft} \times 18{,}810 \text{ ft} \times 0.00325$$

$$= 4.334 \text{ mgd} \times 1.5472 \text{ cfs/mgd}$$

$$= 6.706 \text{ cfs} = 6.7 \text{ cfs}$$

$$Q_d = 1/3 \times T \times L \times I_d = 1/3 \times 70{,}900 \text{ gpd/ft} \times 18{,}810 \text{ ft} \times 0.00325$$

$$= 0.1912 \text{ mgd} \times 1.5472 \text{ cfs/mgd}$$

$$= 0.296 \text{ cfs} = 0.3 \text{ cfs}$$

The analysis was checked using equation 3.

$$Q_m = Q_l + Q_d + \Delta S_m$$

$$7.2 \text{ cfs} = 6.7 \text{ cfs} + 0.3 \text{ cfs} + 0.2 \text{ cfs} = 7.2 \text{ cfs}$$

S_e was computed using equation 8.

$$S_e = \frac{Q_l}{L(h_1 - h_2)} = \frac{6.7 \text{ cfs}}{3.56 \text{ mi } (13.39 - 11.26) \text{ ft}}$$

$$= \frac{6.7 \text{ cfs}}{3.56 \text{ mi} \times 2.13 \text{ ft}} = 0.884 \frac{\text{cfs/mi}}{\text{ft}}$$

$$= 0.9 \frac{\text{cfs/mi}}{\text{ft}}$$

The results indicate that: (1) T equals 70,900 gpd/ft, (2) 6.7 cfs seeped from the lake beneath the Hoover Dike, (3) of the 7.2 cfs measured as discharge from the L-D1 Canal, 6.7 cfs seeped from the lake, 0.3 cfs seeped from the Diston Island Drainage District, and 0.2 cfs was related to depletion of storage in the canal, and (4) the seepage factor was 0.9 cfs per mile per foot of head between the lake and the L-D1 Canal.

TEST 3

The objective of the third test was to determine the seepage beneath the Hoover Dike when the water level in the canal was pumped down several feet below that of the lake. On February 2, Culverts 1 and 1C (fig. 2) were closed and the water level in the canal began to rise (fig. 4) to a static position between the level of the lake and that of the Diston Island Drainage District. Thus, the

gates were positioned so that Pumping Plant No. 1 would only draw water from the 3½-mile reach of L-D1 Canal. On February 6, the C&SFFCD breached the seepage dike beside Culvert 1B and installed Culvert 1Ba with bottom invert at elevation 5.0 feet above msl. During the period of construction (February 6–12), the Diston Island Drainage District intermittently operated an auxiliary pump rated at 22 cfs at Pumping Plant No. 1 and lowered the water level in the canal to about 11.6 feet above msl.

On February 13, the district began to lower the L-D1 Canal by pumping Plant No. 1 at a rate of about 65 cfs. Flow from the canal into the pump bay was measured at Culvert 1Ba. On February 16, the pump lost suction while pumping only 28 cfs. In order to compensate for excessive drawdown and the accumulation of air in the intake it was necessary to increase the pumping rate and circulate some of the discharge back into the pump bay through a nearby gated control.

Water levels in the canal and in observation wells began to approach a steady-state condition by February 19 (figs. 4–7). However, a 0.5-inch rainfall occurred during the night of February 18 and the flow from the canal increased from 26 cfs on February 19 to 30 cfs on February 20. Several more days of pumping would have been required to reach a steady-state condition with no significant benefit to the analysis. Therefore, the pumping test was terminated on February 20 and the data obtained on February 19 at 10:00 a.m. was assumed to be representative of the steady-state condition.

SEEPAGE ANALYSIS

As previously described, an analysis was made using the hydraulic profile at Site No. 1 on February 19, as shown on figure 11, and the basic equations.

Wells 10 and 7 were used to compute I_l, the hydraulic gradient from the lake; and wells 9 and 20 were used to compute I_d, the hydraulic gradient from the district.

$$I_l = \frac{10.44 - 8.89 \text{ ft}}{203 \text{ ft}} = \frac{1.55 \text{ ft}}{203 \text{ ft}}$$

$$= 0.00764$$

$$I_d = \frac{8.90 - 8.36 \text{ ft}}{138 \text{ ft}} = \frac{0.54 \text{ ft}}{138 \text{ ft}}$$

$$= 0.00391$$

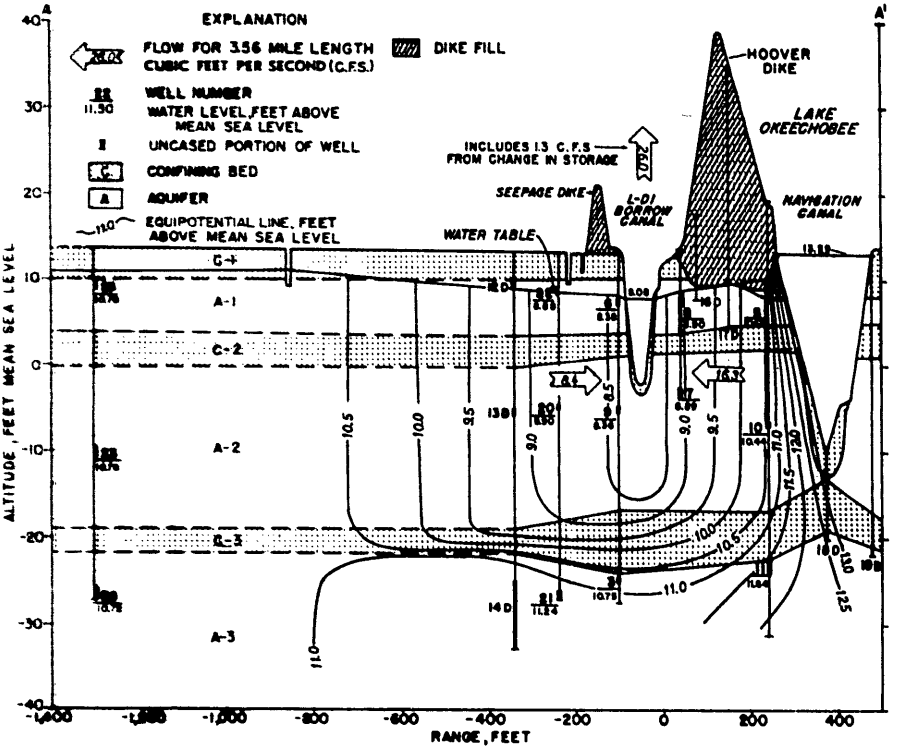

Figure 11. Hydraulic profile along line A-A′ at Site No. 1, Feb. 19, 1968.

The coefficient of transmissibility was computed using equation 7; Q_m was 26.0 cfs; ΔS_m was determined to be equivalent to 1.3 cfs.

$$T = \frac{Q_m - \Delta S_m}{L\,(I_l + I_d)} = \frac{26.0 \text{ cfs} - 1.3 \text{ cfs}}{18{,}810 \text{ ft } (0.00764 + 0.00391)}$$

$$= 0.1137 \text{ cfs/ft} \times 0.64632 \text{ mgd/cfs}$$

$$= 0.07349 \text{ mgd/ft} = 73{,}500 \text{ gpd/ft}$$

The seepages from the lake, Q_l, and from the Diston Island Drainage District, Q_d, were computed using equations 4 and 5.

$$Q_l = T \times L \times I_l = 73{,}500 \text{ gpd/ft} \times 18{,}810 \text{ ft} \times 0.00764$$

$$= 10.56 \text{ mgd} \times 1.5472 \text{ cfs/mgd}$$

$$= 16.338 \text{ cfs} = 16.3 \text{ cfs}$$

$$Q_d = T \times L \times I_d = 73{,}500 \text{ gpd/ft} \times 18{,}810 \text{ ft} \times 0.00391$$

$$= 5.406 \text{ mgd} \times 1.5472 \text{ cfs/mgd}$$

$$= 8.364 \text{ cfs} = 8.4 \text{ cfs}$$

The analysis was checked using equation 3.

$$Q_m = Q_l + Q_d + \Delta S_m$$

$$26.0 \text{ cfs} = 16.3 \text{ cfs} + 8.4 \text{ cfs} + 1.3 \text{ cfs} = 26.0 \text{ cfs}$$

S_e was computed using equation 8.

$$S_e = \frac{Q_l}{L\,(h_1 - h_2)} = \frac{16.3 \text{ cfs}}{3.56 \text{ mi } (13.29 - 8.08) \text{ ft}}$$

$$= \frac{16.3 \text{ cfs}}{3.56 \text{ mi} \times 5.21 \text{ ft}} = 0.879 \frac{\text{cfs/mi}}{\text{ft}}$$

$$= 0.9 \frac{\text{cfs/mi}}{\text{ft}}$$

The results of the analysis indicate that: (1) T equals 73,500 gpd/ft, (2) 16.3 cfs seeped from the lake beneath the Hoover Dike, (3) of the 26.0 cfs measured as discharge from the L-D1 Canal, 16.3 cfs seeped from the lake, 8.4 cfs seeped from the Diston Island Drainage District, and 1.3 cfs was related to depletion of storage in the canal, and (4) the seepage factor was 0.9 cfs per mile per foot of head between the lake and the L-D1 Canal.

The results were re-evaluated using an average hydraulic gradient from the district to the canal based on water-level data obtained from 3 staff gages located in the Diston Island Drainage District along the seepage dike, and from staff gages at Site No. 1, and at the pumping plants, as shown on figure 12.

The average head between the water level at the six observation points in the district adjacent to the seepage dike and the water level in the L-D1 Canal was determined to be 1.12 ft. Because the head at Site No. 1 is related to the hydraulic gradient between wells 20 and 9, the average hydraulic gradient from the district was assumed to be related to the above-mentioned average head adjacent to the seepage dike. Thus, the average hydraulic gradient from the district I_d, was estimated to be 0.00548. I_l was previously computed using the slope between wells 10 and 7.

$$I_l = \frac{10.44 - 8.89 \text{ ft}}{203 \text{ ft}} = 0.00764$$

$$I_d = \frac{\text{Av. head x slope between wells 20 and 9}}{\text{Head between L-D1 Canal and District at Site No. 1}}$$

$$= \frac{1.12 \text{ ft x } 0.00391}{0.8 \text{ ft}} = 0.00548$$

The coefficient of transmissibility was computed using equation 7: Q_m was 26.0 cfs; ΔS_m was determined to be equivalent to 1.3 cfs.

$$T = \frac{Q_m - \Delta S_m}{L\,(I_l + I_d)} = \frac{26.0 \text{ cfs} - 1.3 \text{ cfs}}{18{,}810 \text{ ft } (0.00764 + 0.00548)}$$

$$= 0.1000 \text{ cfs/ft x } 0.64632 \text{ mgd/cfs}$$

$$= 0.06462 \text{ mgd/ft} = 64{,}600 \text{ gpd/ft}$$

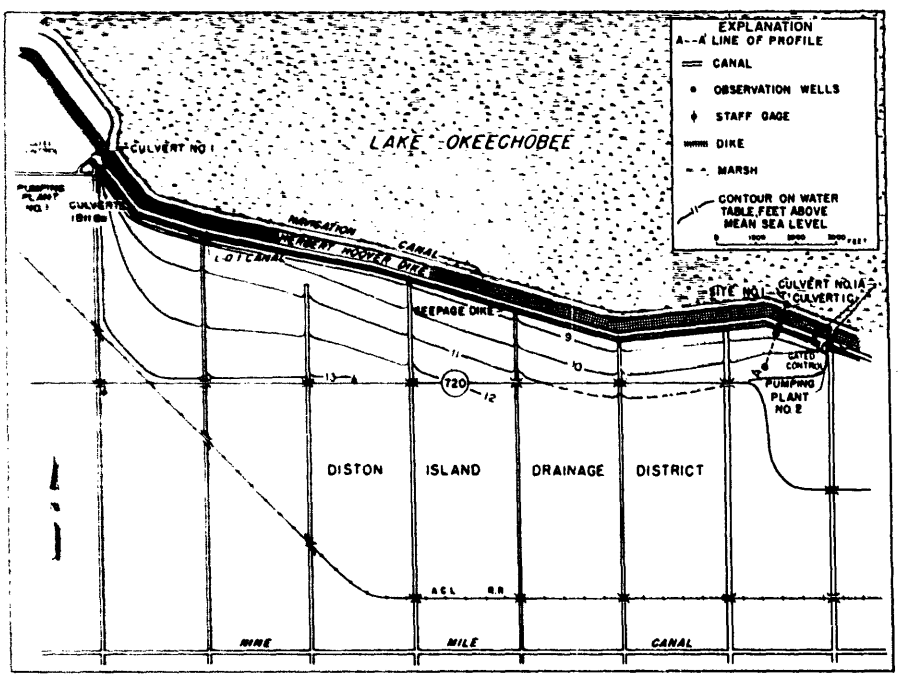

Figure 12. Map showing contours on the water table south of the L-D1 Canal, February 19, 1968.

Q_l, the seepage from the lake, and Q_d, the seepage from the Diston Island Drainage District, were computed using equations 4 and 5.

$$Q_l = T \times L \times I_l = 64{,}600 \text{ gpd/ft} \times 18{,}810 \text{ ft} \times 0.00764$$

$$= 9.285 \text{ mgd} \times 1.5472 \text{ cfs/mgd}$$

$$= 14.37 \text{ cfs} = 14.4 \text{ cfs}$$

$$Q_d = T \times L \times I_d = 64{,}600 \text{ gpd/ft} \times 18{,}810 \text{ ft} \times 0.00548$$

$$= 6.660 \text{ mgd} \times 1.5472 \text{ cfs/mgd}$$

$$= 10.30 \text{ cfs} = 10.3 \text{ cfs}$$

The analysis was checked using equation 3.

$$Q_m = Q_l + Q_d + \Delta S_m$$

$$26.0 \text{ cfs} = 14.4 \text{ cfs} + 10.3 \text{ cfs} + 1.3 \text{ cfs} = 26.0 \text{ cfs}$$

S_e was computed using equation 8.

$$S_e = \frac{Q_l}{L(h_1 - h_2)} = \frac{14.4 \text{ cfs}}{3.56 \text{ mi } (13.29 - 8.08) \text{ ft}}$$

$$= \frac{14.4 \text{ cfs}}{3.56 \text{ mi} \times 5.21 \text{ ft}} = 0.777 \frac{\text{cfs/mi}}{\text{ft}}$$

$$= 0.8 \frac{\text{cfs/mi}}{\text{ft}}$$

Both analyses in this test evaluated T, the coefficient of transmissibility, for a smaller thickness of aquifer than in the other tests. In tests 1 and 2, T was representative of a combined aquifer thickness of about 24 feet while that obtained in test 3 was only representative of about 22.5 feet. Assuming that the permeability is uniform, the value of T obtained in test 3 would have to be adjusted by a factor of 1.06 in order to make a comparison of values obtained in tests 1 and 2.

Therefore, the adjusted T obtained by gradients at Site No. 1 would be 77,900 gpd/ft rather than 73,500 gpd/ft and the other adjusted T would be 68,500 gpd/ft rather than 64,600 gpd/ft.

Because the test did not reach a steady-state condition, it is highly probable that the actual value of T would be less than the corrected values. Therefore, it is assumed that the original computed values are representative of the actual values.

Figure 13 is a composite recovery curve of the water level in the L-D1 Borrow Canal after pumping had ceased. The curve suggests that, if ponded, the water level in the L-D1 Canal would have reached a static level between the stage of the lake and that in the district after about one month of normal recovery. The change in slope of the curve when the water level in the canal exceeded that in the district suggests that seepage into the canal was derived from both the lake and the district.

CONCLUSIONS

The results of test 1 are considered unsatisfactory and are disregarded herein. The results of the tests 2 and 3 are summarized in table 1. Seepage factors ranged from 0.8 to 1.1 cfs per mile per foot of head between Lake Okeechobee and the L-D1 Canal, and coefficients of transmissibility ranged from 64,600 to 92,100 gpd/ft. The range in the computed values is due chiefly to unsteady state conditions during test three (due to rainfall) and to lack of detailed information on hydraulic gradients from the Diston Island Drainage District to the L-D1 Canal during test two. There is, however, a possibility that increased hydrostatic pressure on the filtercake lining the borrow canals reduced the permeability of the filtercake during the tests and thus contributed to some of the variation in values.

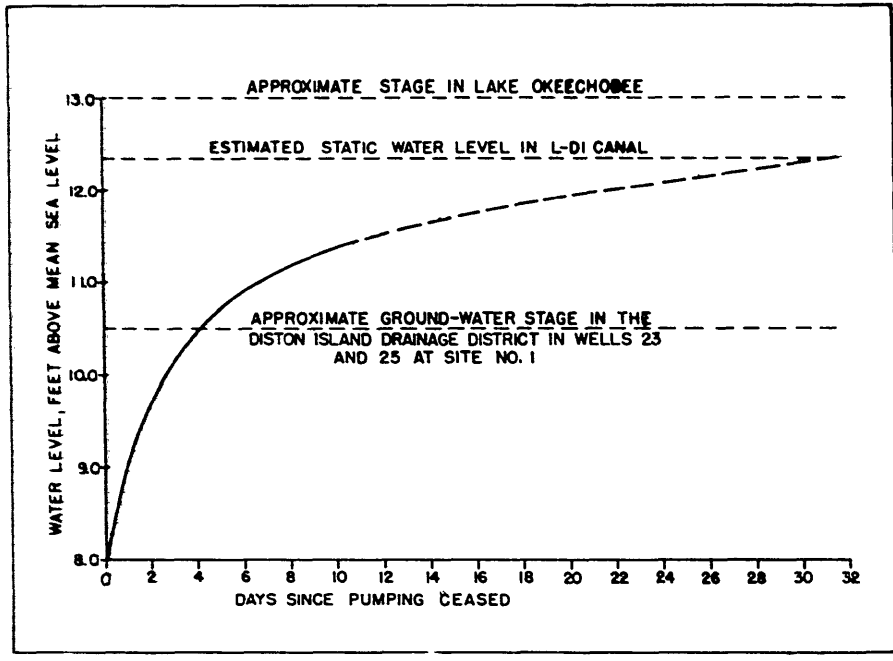

Figure 13. Graph showing recovery curve of water level in L-D1 Borrow Canal, dashed where estimated.

TABLE 1. SUMMARY OF THE SEEPAGE TESTS

Head, Difference between water levels of the lake and the L-D1 Canal, feet; D, Discharge from L-D1 Canal measured at either Culvert 1B or 1C, cfs; S_e, Seepage factor, cfs per mile of L-D1 Canal per foot of head between the canal and the lake; and T, Coefficient of transmissibility, gallons per day per foot.

Results based on hydraulic gradients at Site No. 1:

Date	Head, feet	D cfs	Seepage (cfs) From Lake	From District	To District	S_e	gpd/ft T
01-29-68	2.14	8.3	8.6	—	0.8	1.1	92,100
02-02-68	2.13	7.2	8.1	—	1.1	1.1	85,300
02-19-68	5.21	26.0	16.3	8.4	—	.9	73,500
Results based on hydraulic gradients at Site No. 1 adjusted for higher water levels in the western part of the Diston Island Drainage District							
01-29-68	2.14	8.3	7.6	.4	.2	1.0	81,300
02-02-68	2.13	7.2	6.7	.6	.3	.9	[1]70,900
02-19-68	5.21	26.0	14.4	10.3	—	.8	[1]64,600

[1]not adjusted to complete saturation.

The adjusted results of tests 2 and 3 (table 1) are considered to be most representative of the values of T and S_e. Thus, estimates of seepage in the area may be made using the average of 72,300 gpd/ft for the coefficient of transmissibility and 0.9 cfs per mile per foot of head between the lake and the L-D1 Canal, as the average seepage factor. For example, when the water level of the lake is raised to 19 feet above msl and the water level of the canal is lowered to 11 feet above msl, the seepage from the lake is the product of the seepage factor (0.9 cfs per mile per foot of head between the lake and the L-D1 Canal) and the loss in head (8 feet) or 7.2 cfs per mile.

The seepage landward of the L-D1 Canal is related to the operational stage of the canal. The canal can be used to intercept almost all or part of the seepage from the lake, depending on the stage of the canal and the desired (regulated) ground-water level in the Diston Island Drainage District adjacent to the canal.

If the stage in the canal is maintained at, or slightly below, the stage of the water table in the district then significant amounts of seepage from the lake will be intercepted by the canal. If the stage in the canal is maintained below the stage of the lake but above that in the district then the seepage into the district will be proportional to the total seepage from the lake and to the discharge from the canal. The seepage into the district can be estimated by (1) subtracting the canal discharge from the estimated seepage beneath the Hoover Dike, or by (2) computing the seepage using Darcy's law, the hydraulic gradient into the district, and the coefficient of transmissibility (72,300 gpd/ft).

The distance beyond which no effects will occur due to raising the level of the lake is governed by the occurrence of ditches or canals which act as hydraulic boundaries. For instance, during Test 3 the water levels in wells 23–25 located 1,200 feet south of the canal were apparently unaffected by the drawdown in the L-D1 Canal because of a constant-head recharging field ditch.

The simplest way to control the inland effects of raising the level of the lake is to maintain the water level in the L-D1 Canal at, or slightly below, the desired water level in the fields adjacent to the seepage dike (fig. 3). Therefore, the effects of raising the level of the lake will be chiefly limited to water levels in aquifers A-1 and A-2 beneath the Hoover Dike. However, there will be a corresponding rise in the artesian pressure in aquifer A-3 beyond the L-D1 Canal which will produce a slight increase in leakage through confining bed C-3. At present, this leakage is considered negligible but could become a significant factor if canals are dug deep enough to penetrate bed C-3.

If the water level in the L-D1 Canal is maintained above that in the adjacent fields then the water level in the L-D1 Canal will affect the water levels in the adjacent fields in the same manner as the lake. The inland extent of the effects of raising the water level in the L-D1 Canal above that in the adjacent fields will be chiefly limited by the occurrence of a discharging field ditch or canal at a constant head. If the loss in head across the silt lining the L-D1 Canal is neglected then the slope of the piezometric surface in aquifers A-1 and A-2 would approach a straight line from the water level in the L-D1 Canal to the water level at the nearest discharge boundary. There will also be a corresponding rise in the piezometric surface in aquifer A-3 so that increased upward leakage through bed C-3 may occur.

SUMMARY

The results of the tests on the L-D1 Borrow Canal indicate that the seepage factor is about 0.9 cfs per mile per foot of head between the lake and the canal, and that the coefficient of transmissibility is about 72,300 gpd/ft. These results compare favorably with those previously determined by F. W. Meyer in 1966. Seepage from the lake into the L-D1 Canal is estimated to be on the order of 7.2 cfs per mile when the stage of the lake is 19.0 feet above mean sea level and the stage of the L-D1 Borrow Canal is 11.0 feet above mean sea level. Most of the seepage will occur through aquifers A-1 and A-2. Seepage losses through aquifer A-3 are considered negligible.

Seepage landward of the L-D1 Canal is related to the operational stage of the canal. The canal can be used to intercept almost all or part of the seepage from Lake Okeechobee depending on the ground-water levels desired in the fields adjacent to the canal. If the stage in the canal is maintained at, or slightly below, the stage of the water table in the fields then the canal will intercept all significant amounts of seepage.

The significance of the tests lies not in the precision of the computed seepage factors and coefficients of transmissibility but in their magnitude and their relationships to the geology of the area. The reported values of T and S_e may be used to estimate seepage rates in other areas having similar hydrogeologic conditions, and should result in significant savings in the design and operation of flood-control works around Lake Okeechobee.

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